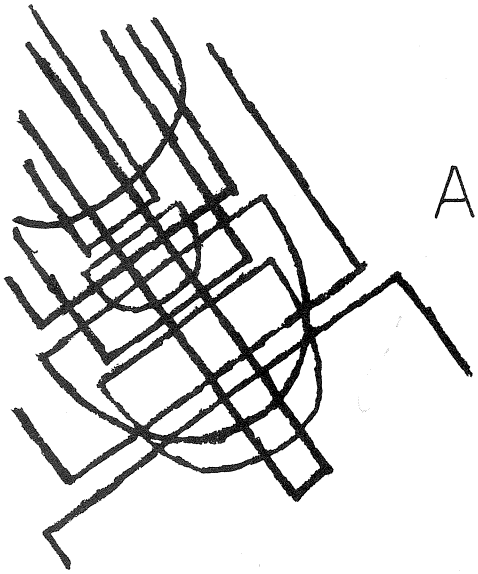

A STATIC AND DYNAMIC PILE TEST

IN POCATELLO, IDAHO

DIVISION OF SOLID MECHANICS, STRUCTURES AND MECHANICAL DESIGN
CASE WESTERN RESERVE UNIVERSITY
CLEVELAND, OHIO

A Report on

A STATIC AND DYNAMIC PILE TEST

in

POCATELLO, IDAHO

Performed on

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in Cooperation with the

Idaho Highway Department

and the

Federal Highway Administration

by

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Division of Solid Mechanics, Structures and Mechanical Design

Case Western Reserve University

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Acknowledgements

The measurements reported here and their analysis was sponsored by the Idaho Department of Highways and the Federal Highway Administration. The cooperation and assistance of Messrs. L. F. Erickson, W. V. Jones and W. F. Mares all of the Idaho Department of Highways in arranging and conducting the tests is gratefully acknowledged.

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Forward

The work presented here is a part of a larger research activity on dynamic pile behavior at Case Western Reserve University. Most of this work has been sponsored by the Ohio Department of Highways in cooperation with the Federal Highway Administration. In order to increase the amount of data available the cooperation of other highway departments has been sought in making available, for dynamic testing, piles on which static load tests were to be conducted.

The Idaho Department of Highways prepared a report describing their soil investigation, foundation design and pile load tests. Since this report is not readily available but very useful to the reader copies can be made available by the Case Western Reserve University. The authors believe that the Idaho report adds considerably to the usefulness of their report and appreciate the cooperation of the Idaho Department of Highways in making it available.

1. Summary

Dynamic tests were made on three piles at a site located about 15 miles west of Pocatello, Idaho. A complete static load test was available for only one of these piles (Pile No. 11). The results of the tests on the piles which were not statically load tested are presented in the detailed portion of this report. The test pile was a 16-inch diameter cast-in-place concrete pipe pile with a 3/8-inch wall thickness. It was driven, with a closed end, through seven feet of sand and gravel and 20 to 30 feet of medium stiff silty clay into a loose sand. It was expected that the bearing capacity of the sand could be improved by densification during the driving of this displacement type pile. All tests were performed prior to filling the pile with concrete.

After completion of the standard Idaho load test a constant rate of penetration test was performed by Case Western Reserve University project personnel. An ultimate capacity of 390 kips was reached. The results of this test together with the Maintained Load Test performed by the Idaho Department of Highways is shown in Fig. 2. After completion of the load test the pile was struck with a Link-Belt 312 hammer. Dynamic measurements were made in three approximately equal sets of about 55 blows each. Total permanent sets of 1.0, 1.25 and 2.75 inches were obtained for each set of 55 blows, indicating a gradual loss of set-up strength at the soil-pile interface. Dynamic measurements of force and acceleration were recorded on a magnetic tape recorder.

The dynamic data was processed using a system developed at Case Western Reserve University. The total analog tape record was played into an analog-to-digital converter which was connected to a small digital computer. The blow records were stored and the simplified predictions of static capacity described in Reference 2, were performed in real time. The results of these computations are presented in Figure 7. The low

predictions near blows 56 and 112 are due to the small energy delivered during stopping and restarting the hammer. The loss of resistance capacity due to set-up is clearly indicated. The maximum Phase IIA prediction is 317 kips, the average for the first ten blows is 289 kips and the average for the last ten blows is 264 kips. The most meaningful correlation for evaluating this prediction is to compare it with the load test result at comparable displacements (Ref. 2). This can be best clarified with an example. The average maximum dynamic displacement for the first ten blows, as obtained by double integration of the acceleration, was 0.20. The load associated with this displacement in the C.R.P. test is found to be 315 kips as shown in Figure 2. If the dynamically predicted capacity is compared with this static value the differences are 0.6% and 8.3% for the maximum value and the first ten, respectively. The differences between the dynamically predicted values and the ultimate is 19% and 26%, again for the maximum and first ten, respectively.

A prediction of soil resistance distribution was made using the computer program developed at Case and described in Reference 2. Selected digitized records obtained from the analog-to-digital conversion activity mentioned above were transferred by punched paper tape to a Univac 1108 computer. The results of analysis are presented in Fig. 9. The resistance distribution at ultimate load is presented by showing the distribution of the force in the pile. Also shown is the prediction made by the Idaho Department of Highways (Reference 1) based on the analysis of soil properties.

As an example for the interpretation of pile force distribution graphs consider Figure 9 and, in particular, the prediction from blow No. 2-A (The A refers to the fact that the records were made after a set-up period). This plot shows a pile top force of 301 kips which is the total predicted capacity. At a depth of 25 feet the force in the pile was 206 kips indicating that a resistance force of 301-206 = 95 kips acts over the upper 25 feet of the pile. This value is higher than found for the broken anchor pile that developed a static friction resistance of 70 kips. It should be mentioned in this context that the results plotted in Figure 9, 11 and 13 contain some numerical uncertainties which can be eliminated by smoothing the pile force distribution curves. This would probably lead to even better agreement with the friction force measured on the broken anchor pile.

It is of interest to observe the change in resistance distribution with driving. As the blow count increases the tip resistance increases but that action is more than canceled by the reduction of skin resistance. The total static capacity predicted by the wave analysis was the largest for blow number two with a magnitude of 301 kips.

These tests provided very valuable additional data for evaluating dynamic pile test methods. Also the capability of the project for obtaining dynamic measurements on short notice at considerable distance was thoroughly tested. The results are judged to be satisfactory. Recently developed simplified methods, not yet reported and not included in this report, show even better correlation with ultimate capacity.

2. General Information

2.1 Pile Details

The test site was located west of Pocatello, Idaho, at the intersection of Gasline Road with the proposed I-15W-4(12)81.

Three piles were selected for dynamic testing. Pile No. 19 was an open ended 16-inch diameter pipe pile with 3/8-inch wall thickness. Pile No. 11 was a closed end pipe having the same properties as No. 19. Both piles were driven to a depth of 44'-3". A third pile was a H pile that served as an anchor pile during the static load test on pile No. 11. The layout of the piles is given in Figure 1 which was taken from Reference 1.

2.2 Soil Investigation

Soil borings conducted by the Idaho Highway Department (IHD) showed that the test piles were driven through about 7 feet of sand and gravel and 20 to 30 feet of medium stiff silty clay and clay into loose sand. A soil profile taken from Reference 1 is shown in Figure 9. Other soil investigations were undertaken and are described in Reference 1.

2.3 Driving Information

The piles were driven and restruck with a Link-Belt 312 hammer having a rated energy of 18000 ft-lb. Piles No. 11 and 19 were both driven to a depth of 44'-3". At this depth the closed ended pile No. 11 exhibited a blow count of 52 blows per foot. The open ended pile (No. 19) showed a higher resistance (92 blows per foot). Using the EN Formula, design load predictions of 84 and 109 kips are obtained for piles No. 11 and 19, respectively. Theoretically, these design loads contain a safety of six which means that failure loads of 504 and 654 kips are expected for the two piles under consideration. The H-pile reached a blow count of 56 blows per foot which corresponds to a design load of 92 kips. It should be noted, however, that other safety factors are commonly used depending on experience with particular soil and hammer types.

2.4 Static Load Tests

Prior to dynamic testing the IHD had performed Maintained Load (M.L.) tests on both pile No. 19 and No. 11. Specifications for these M.L. tests are given in Reference 1. Two test cycles were attempted on pile No. 19 but were not completed because of jack and anchor pile failure. Pile No. 11 was subjected to two successful test cycles in addition to a constant rate of penetration test (C.R.P.) performed by Case Western Reserve

University project personnel. The C.R.P. test was performed immediately after the second M. L. test cycle was complete. A displacement rate of 0.02 inches per minute was used during the early part of the test and later the rate was increased to 0.05 inches per minute. All three test results are plotted in Figure 2.

Pile No. 4 (H-pile) served only as an anchor pile and was not load tested statically.

3. Dynamic Measurements

3.1 Data Acquistion

After the C.R.P. test was completed on pile No. 11 the reaction beam and jack were removed and transducers were attached to pile No. 19. The instrumentation for the dynamic recordings is schematically shown in Figure 3. In addition to the strain transducers shown in Figure 3, a second set of clip-on transducers was attached as a back-up system. Further illustration of the transducer and recording system is shown in Figures 4 and 5.

Two acceleration and two strain records were obtained on pile No. 19 under each of approximately 220 hammer blows with a total pile penetration of 2.25 inches.

After completion of the dynamic test on pile No. 19 the transducers were mounted to the wall of pile No. 11. Three different sets of records were obtained with changes in recorder calibrations since peak acceleration values exceeded the expected maximum value of 500 g's for some of the blows in the first set.

Each of the three sets of records consisted of approximately 55 blows yielding penetrations of 1.0, 1.25 and 2.75 inches. The progressive gain in penetration indicated a loss of set-up strength at the pile-soil interface due to driving effects.

A third dynamic data set was obtained from pile No. 4 (12 BP 53). This time only one set of clip-on transducers was used. Again, three sets of blows were recorded. For the first 60 blows the pile penetration was 1.25 inches. A set of 3.0 inches was obtained for the next 70 blows and 5.25 inches under the last 86 blows.

All blows were numbered consecutively for each pile starting with 1-A for the first blow. The A indicates that the records were taken after a set-up period.

3.2 Data Processing

As a first check on the accuracy of the dynamic data a few records were reproduced by playing the magnetic tape into an oscillograph. The two acceleration and the two strain records were inspected to see whether their behavior was in agreement. A sample record is shown in Figure 6. (Note, this record is uncalibrated and merely represents an image of the magnetic tape record). Next the magnetic tape was played into an analog-to-digital converter which was connected to a small digital computer that stored the data of one blow and

a. provided a quality control of the individual blow record by display of the digital record on an oscilloscope,

b. performed the computations necessary to obtain simplified predictions of static bearing capacity,

c. punched the record on paper tape which is compatible with a large electronic computer. In this way the data can be further analyzed.

All analog records on magnetic tape were used for simplified predictions.

4. Dynamic Predictions of Static Capacity

4.1 Simplified Methods

A number of simplified methods for predicting pile capacity have been proposed during the Ohio Department of Highways research project. In the analysis of this data the method referred to in Reference 2 as Phase IIA was used. The results of the data analysis are given in Table 1. It was possible to obtain simplified predictions for all blows recorded.

Figure 7 shows a graph of simplified prediction vs. blow number for pile No. 11. There is a clear indication of a loss of capacity during driving. A maximum capacity of 317 kips was predicted for blow No. 6-A. An average prediction of 289 was obtained for the first 10 blows while the last 10 blows showed an average resistance of 264 kips. This shows the disintegration of the setup strength probably coming from the silty clay layers. Low predictions occurred near blows No. 56 and 112 due to either stopping or restarting the hammer.

It should be noted that pile No. 19 did not show the same characteristics of strength loss as pile No. 11. This phenomina will be discussed further below.

4.2 Wave Analysis Methods

4.2.1 Pile No. 11

The results presented in this section were obtained by a method which is referred to as wave analysis and is explained in detail in Reference 2.

This pile was especially useful for analysis because of its complete static load test results. Records of three blows were analyzed to predict the distribution and magnitude of static and dynamic resistance forces. Predictions of maximum static and dynamic resistance forces are summarized in Table 2. These results will be discussed further in Section 4.3 for all dynamically tested piles.

In the analysis the piles was divided in ten elements, each about 4.3 feet long. Using the measured acceleration as an input, soil resistance forces were predicted such that the computed pile top force matched the measured force. The results for the second blow are shown in Figure 8. Also, the measured velocity is plotted after multiplication by the proportionality constant EA/c (where E is Young's Modulus, A is the cross sectional area and c is the wave speed of the pile). The match shown is good, thus, the analysis results can be considered reliable within the limits of the soil model employed (a linear damper and an elastic-plastic shear resistance law, see Reference 2). The pile force distributions for the three blows under discussion are shown in Figure 9. Also plotted in this Figure is the IHD static prediction which was based on the soil investigation and is given in Reference 1, Figure 4. Note that forces in the pile are shown as they would be measured during a static load test. Another comparison is possible using the knowledge of skin friction from the upper 25 foot portion of the broken anchor pile reported in Reference 1. It supported an uplift load of 70 kips. The graph for blow No. 2-A in Figure 9 shows a somewhat larger resistance of 90 kips over that depth, while blow No. 7-A predicted somewhat smaller value of 55 kips.

While the total pile capacity decreases with increasing blow number a gain in pile toe force is found and all the loss of resistance force occurs along the skin. This clearly indicated tendency has a very reasonable explanation. The soil around the pile tip was densified during, and relaxed after the driving operation, thus, exhibiting only a relatively small resistance at the onset of redriving. During restriking the sand layer gained strength due to further densification. On the other hand the silty clay around most of the pile shaft could be expected to produce set-up strength. Most of this additional skin resistance was apparently lost after few hammer blows. Figure 9 confirms these observations. Fairly good agreement is observed between the IHD prediction based on soil mechanics and the prediction for blow 2-A. It is interesting to observe that the dynamic prediction of skin resistance is essentially zero over the upper 15 feet.

4.2.2 Pile No. 19

This pile was driven open ended. A soil plug moved up the pile to approximately 30 feet above the pile bottom. The presence of this additional mass inside the pile makes a dynamic analysis subject to errors since little is known about the effective inertia and elastic properties of the plug. Also, little is known about the mechanism at the plug-pile interface. Assuming, however, a similar behavior between the soil and the pile both inside and outside allows the prediction of soil distribution in the usual manner, implying that the presence of the plug is neglected.

Two blows (No. 35-A and No. 132-A) were analyzed. Earlier blows could not be used since no permanent set was obtained before blow 35-A because of the high pile resistance. Also other blows showed very little permanent set in the dynamic records.

The match for measured and predicted force is shown in Figure 10 for blow No. 35-A. Note that there is a much higher impact force than in the case of pile No. 11, indicating a high soil resistance which returns large portions of the driving energy back to the hammer.

Figure 11 shows the predicted resistance distribution for both analyzed blows. Again a strength gain at the pile toe can be observed between the earlier and the later blows. However, there was only a small loss of skin friction so that the overall capacity even increased slightly.

4.2.3 H-Pile

Several records, taken on the H-Pile (No. 4), were analyzed. Figure 12 shows a match of dynamic measurements and calculated values for this pile and Figure 13 gives the resistance distribution for the two blows analyzed.

There was an effect of redriving on total capacity and resistance distribution. Note, however, that the pile was longer and penetrated deeper into the sand layer where it apparently obtained most of its strength. The strength losses during redriving again occurred in the clay layer but there was little strength gain at the tip, a behavor that might be expected since the H-pile does not displace as large a volume as the pipe pile.

4.3 Discussion of Results

Predictions from wave analysis are summarized in Table 2 with the maximum damping forces encountered under the blow. The predictions from wave analysis are lower than those from simplified predictions. This can be attributed to relatively large damping forces (see Table 2) found by the wave analysis which introduces errors into the simplified analysis. For pile No. 11 it is also possible to compare the load vs. penetration curve from static load tests with a similar curve from a static analysis using the predicted resistance distribution. This comparison is shown in Figure 14. Note that the deflection scale in Figure 14 is four times smaller than in Figure 2 allowing differentiation of the four graphs. There is a very good correlation between the C.R.P. test curve and the result from blow No. 2-A up to the point when the predicted ultimate capacity is reached.

Two reasons can be given for the predicted capacity being lower than determined in the static load test.

(i) The displacement of the top of pile No. 11 was determined by double integration of the acceleration record. The maxima of these displacements were between 0.17 and 0.24 inches. These displacements are associated with forces of 270 to 360 kips on the load test curve. Such small displacements are not sufficient to overcome the ultimate capacity of the pile (the C.R.P. test reached its ultimate at about 0.28 inches pile top penetration).

(ii) It was found that dynamic effects decrease the strength in the clay-pile interface while the sand became compressed during driving. Pile No. 19 was tested under 220 blows. Piles No. 11

> and 19 were only six feet apart. Thus, dynamic effects might have affected the skin resistance of pile No. 11 during the redriving of No. 19 while the toe force did not change.

As observed already from the two wave analysis results, pile No. 19 did not show any loss of overall capacity since losses at the skin were compensated by gains at the toe. Similarly, the simplified methods did not predict differently between the first and the last blows. A maximum value of 454 kips was found during the first ten blows. Wave analysis predictions were somewhat higher. It should be noted again that the pile penetration under most of the blows was very small so that the results probably indicated lower bounds of the pile capacity. A predicted load vs. deflection curve for this pile is given in Figure 15.

H-pile results are listed in column 3 of Table 1. The capacity of this pile is only slightly below that of pile No. 11. Its greater length apparently compensated for the smaller surface area. The predicted load vs. deflection curve is also given in Figure 15.

The results of these tests are very interesting to compare. It might have been expected that pile No. 11 would have shown a higher capacity than pile No. 19 as it was driven with a closed end. Since the plug in No. 19 extended over a substantial length of the pile less densification would be expected in the sand. No. 19 did not lose strength during redriving while a substantial loss of strength occurred in No. 11. The dynamic analysis of this fact is confirmed by the redriving record. Pile No. 19 did not exhibit the large change in blow count that was observed in Pile No. 11. The H-pile located nearest to No. 11 exhibited an even greater loss of resistance on redriving. It seems likely that the above differences can best be explained by variation in soil characteristics over even the limited area involved. This emphasizes the value of a reliable inexpensive test procedure.

5. Summary

Dynamic data were successfully obtained on three piles. Simplified and wave analysis methods were used to predict pile strength and static soil resistance distribution. It was found that driving effects decreased the skin resistance in the clay layer and increased the pile point capacity. Correlation between static load test and dynamic prediction was possible only for pile No. 11. It was found that the dynamic methods predicted below the statically measured ultimate but consistent with the static values at displacements comparable to those induced during driving. It is possible that the resistance of pile No. 11 was disturbed by restriking pile No. 19. However, it is safe to conclude that the open ended pile had a higher capacity than the closed end pile. A theoretical load vs. penetration curve was determined by static analysis utilizing dynamically predicted soil parameters.

6. References

1. Jones, W. V., and Mares, W. F., "Pile Driving and Pile Loading Tests conducted on Steel Shell and Steel 'H' Piles for Project I-15W-4(12)81 Igo O.P. To Bannock Creek, Power County, Idaho", Report distributed by the Idaho Highway Department, May 1970.

2. Goble, G. G., Rausche, F., and Moses, F., "Dynamic Studies on the Bearing Capacity of Piles", Final Report, Phase 3, Case Western Reserve University, to be published.

TABLE 1

CAPACITY PREDICTIONS FROM THE SIMPLIFIED METHOD

Column	1	2	3
Blow	Pile No. 19 Total Static Capacity	Pile No. 11 Total Static Capacity	Pile No. 4(H) Total Static Capacity
Maximum	454	317	289
Avge of first ten blows	419	289	260
Avge of last ten blows	422	264	245

TABLE 2

CAPACITY PREDICTIONS FROM WAVE ANALYSIS

Pile No. 19			Pile No. 11			Pile No. 4(H)		
Blow No.	Total Static	Maximum Damping	Blow No.	Total Static	Maximum Damping	Blow No.	Total Static	Maximum Damping
35-A	466	250	2-A	301	86	9-A	292	114
132-A	489	190	7-A	257	174	78-A	265	99
			152-A	221	140			

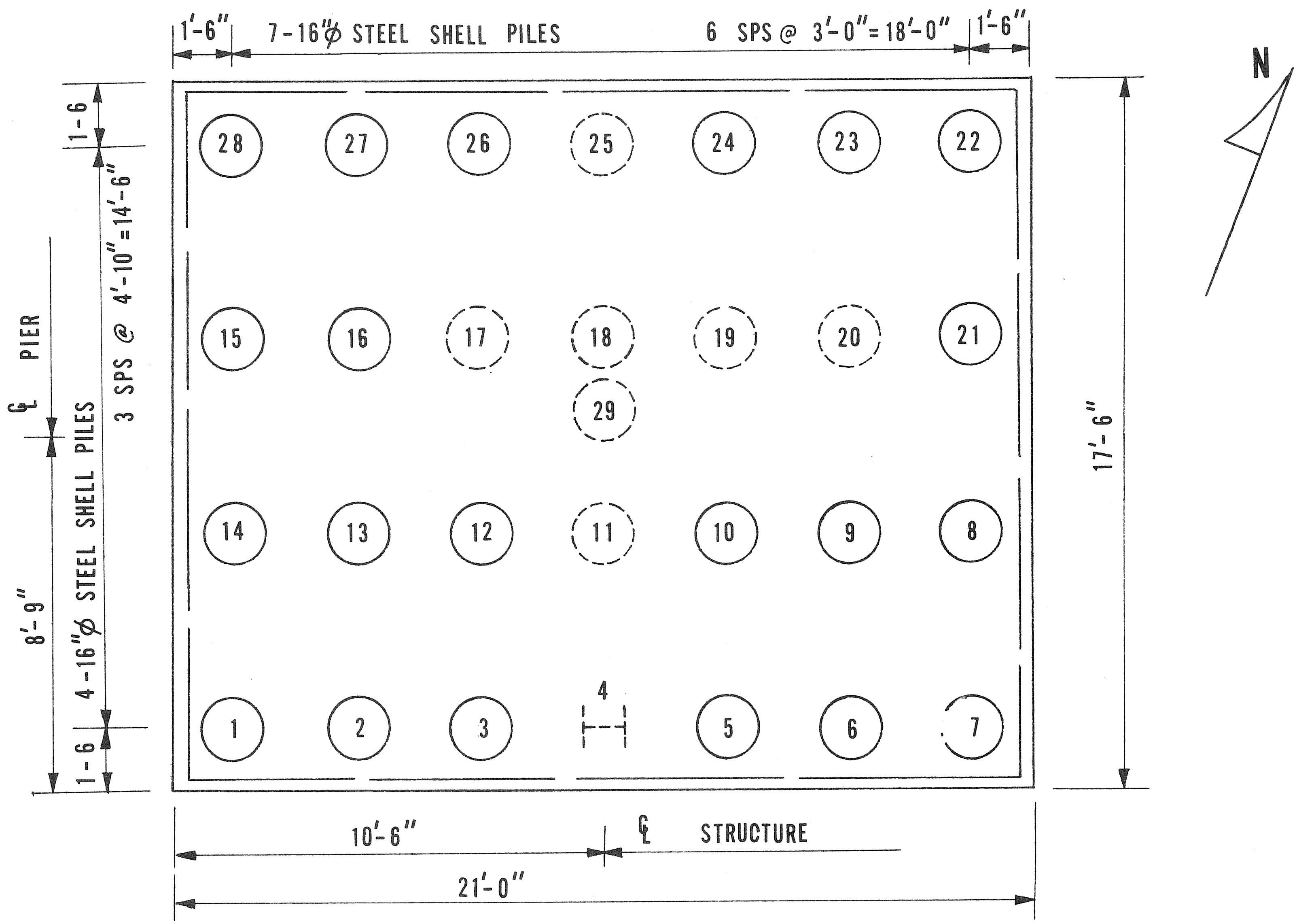

FIGURE 1: LAYOUT OF TEST AND ANCHOR PILES

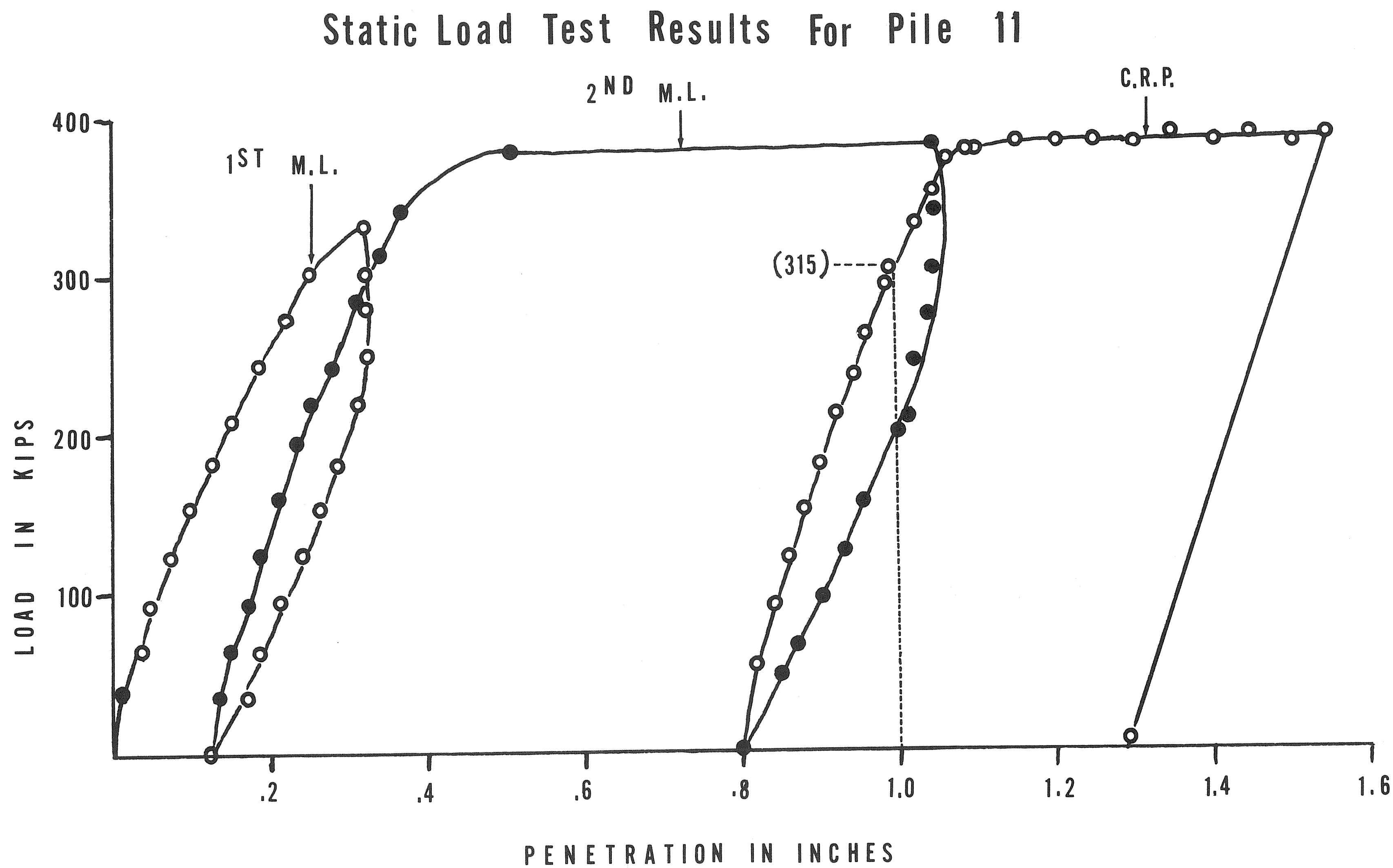

FIGURE 2: LOAD VS. PENETRATION CURVES FOR STATIC TESTS LOADS PERFORMED ON PILE NO. 11

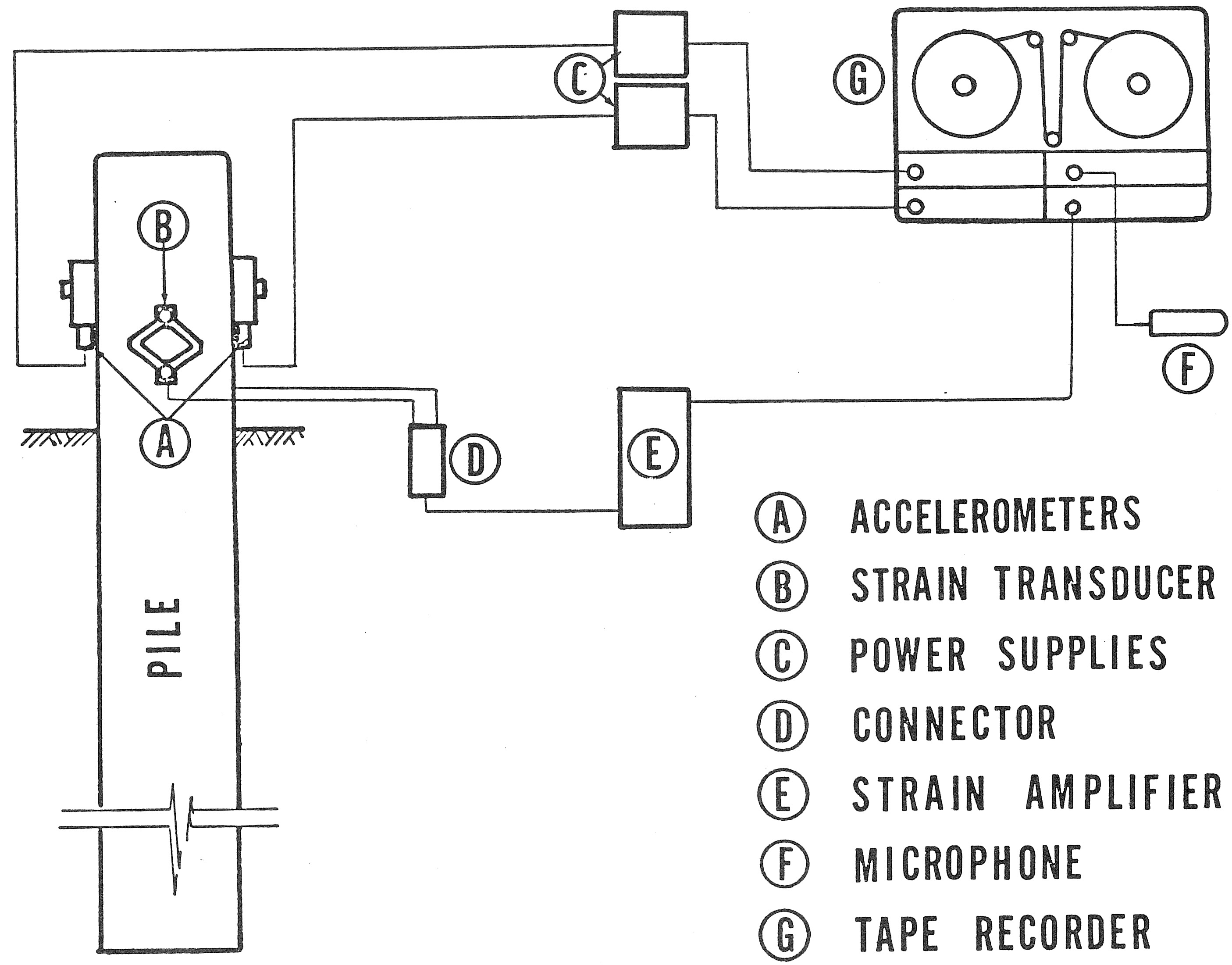

FIGURE 3: SCHEMATIC OF RECORDING SYSTEM

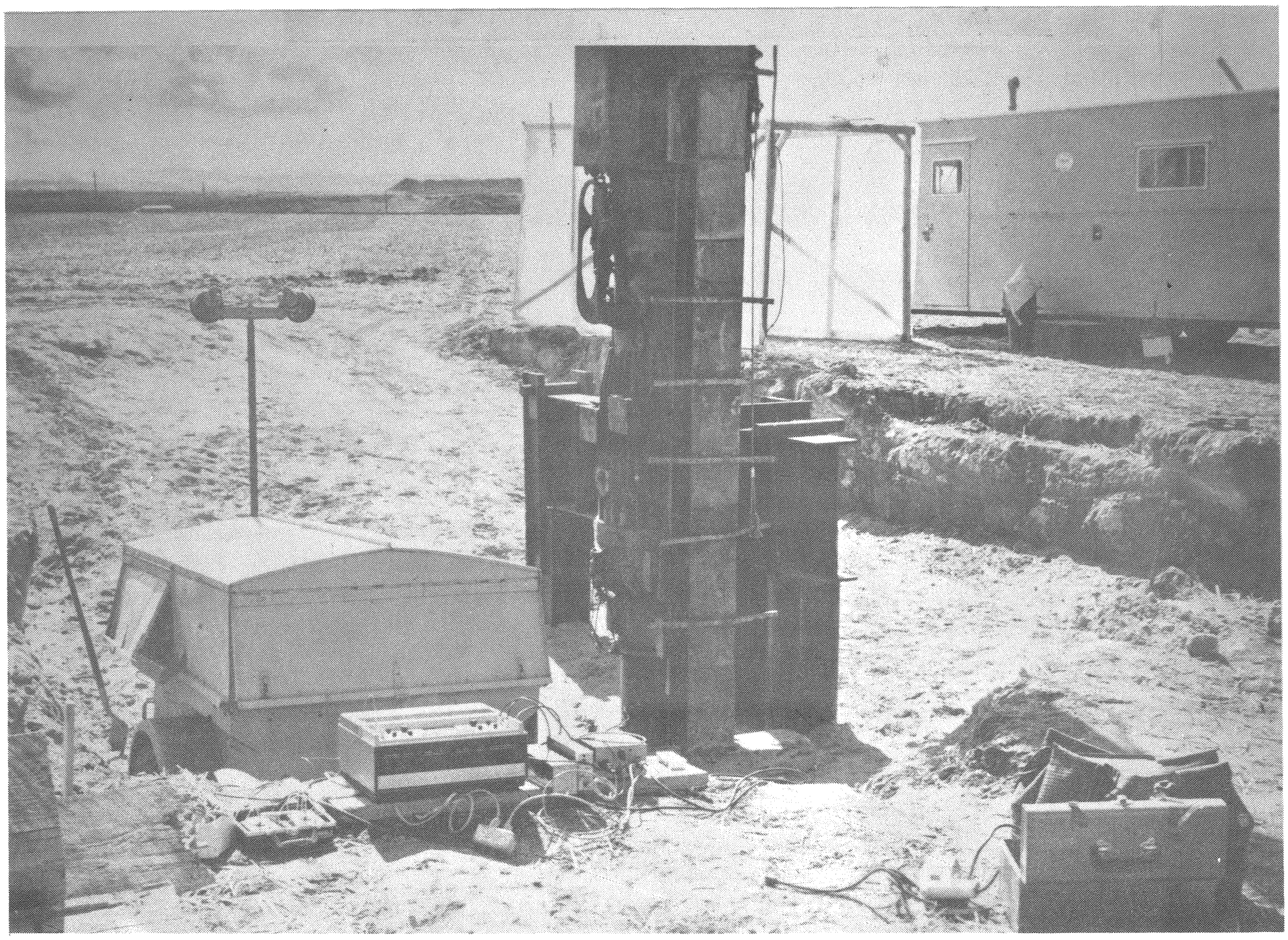

FIGURE 4: RECORDER AND SIGNAL CONDITIONING EQUIPMENT

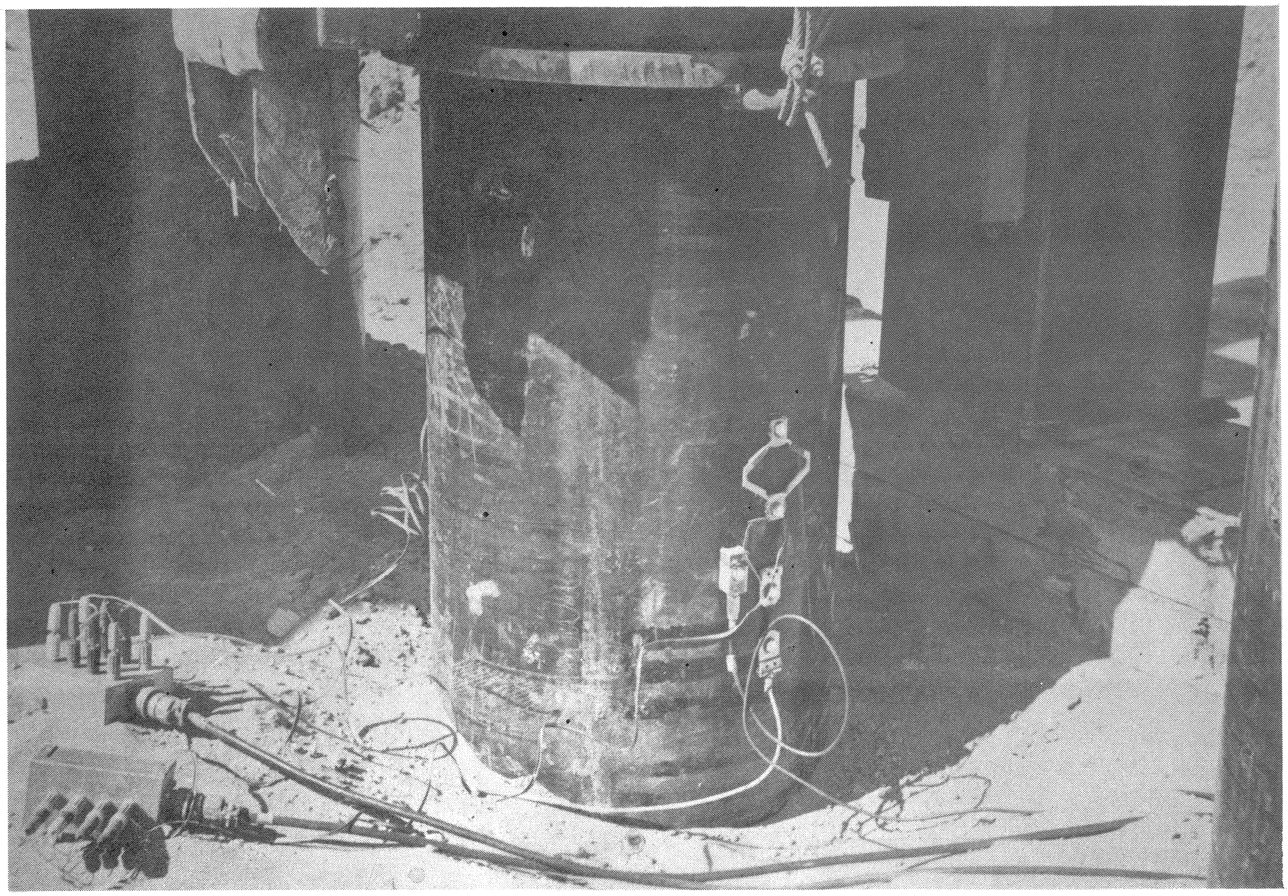

FIGURE 5: ACCELEROMETER AND TWO STRAIN TRANSDUCERS ATTACHED TO ONE SIDE OF PILE. (AN IDENTICAL SET OF TRANSDUCERS IS ATTACHED AT 180°)

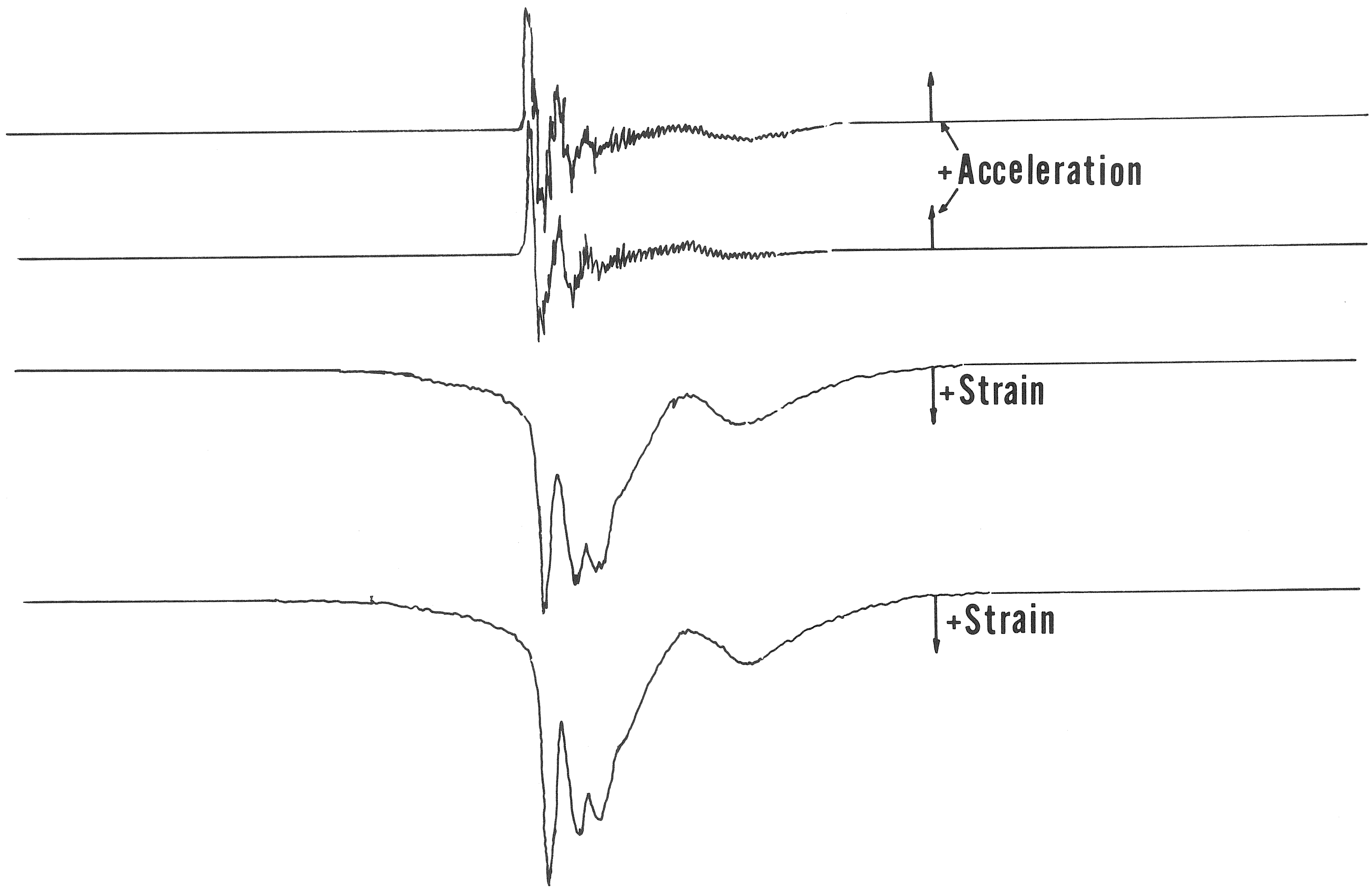

FIGURE 6: UNCALIBRATED DIRECTLY REPRODUCED DYNAMIC RECORD

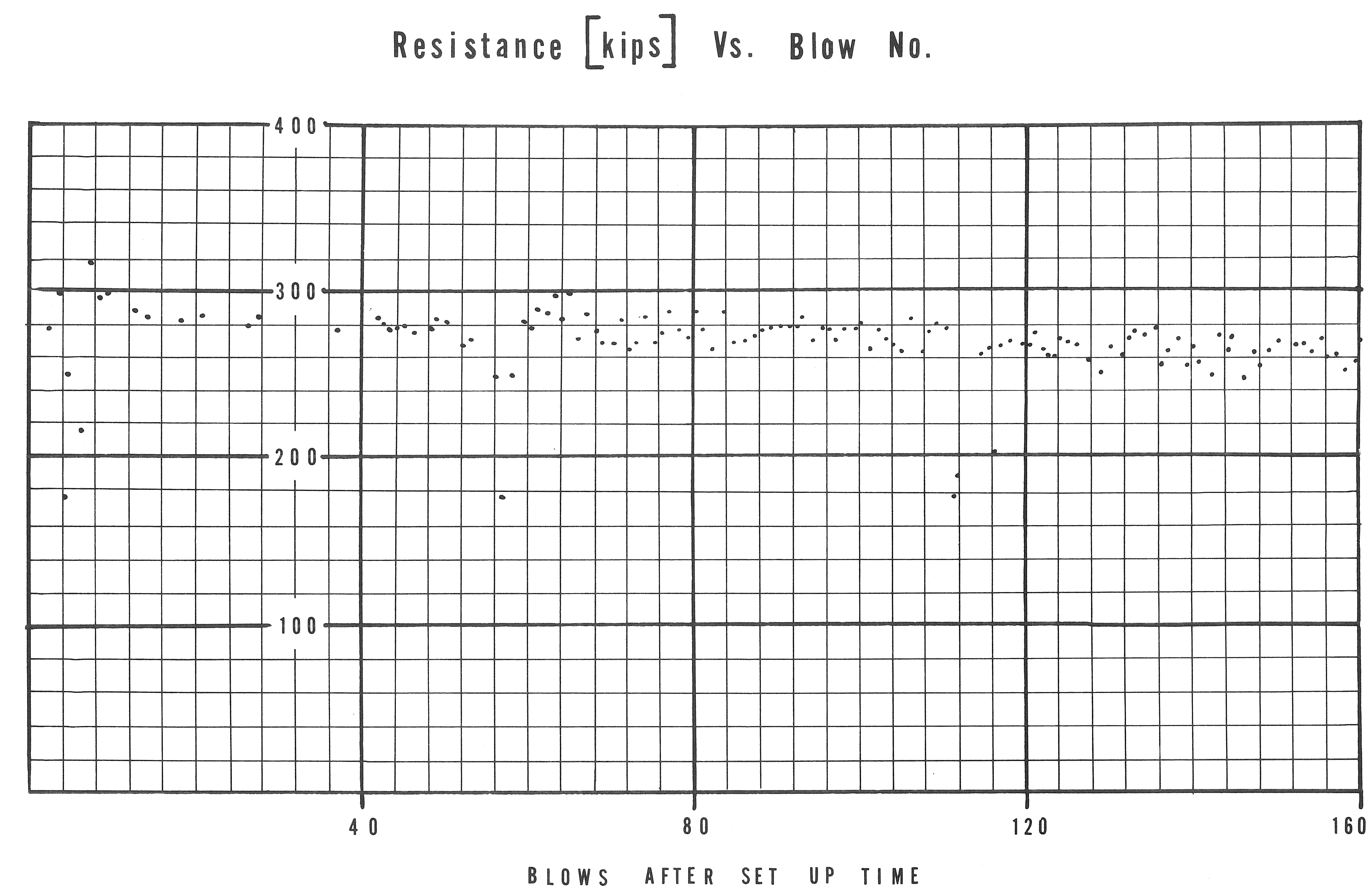

FIGURE 7: RESISTANCE VS. BLOW NUMBER FOR PILE NO. 11 (USING A SIMPLIFIED METHOD).

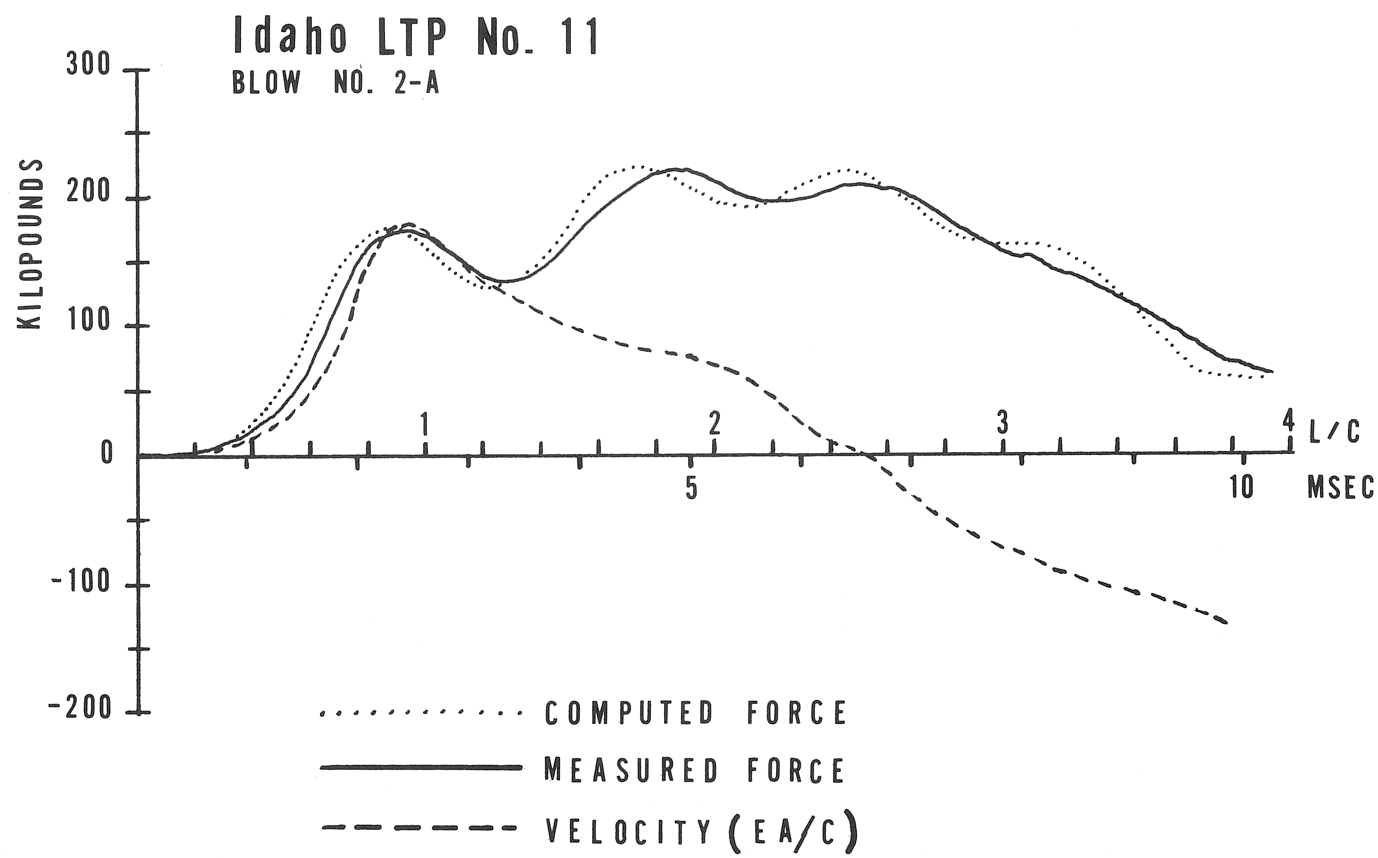

FIGURE 8: COMPARISON OF COMPUTED WITH MEASURED FORCE AND MEASURED VELOCITY FOR PILE NO. 11, BLOW NO. 2-A.

Predicted Pile Force Distribution For Pile 11

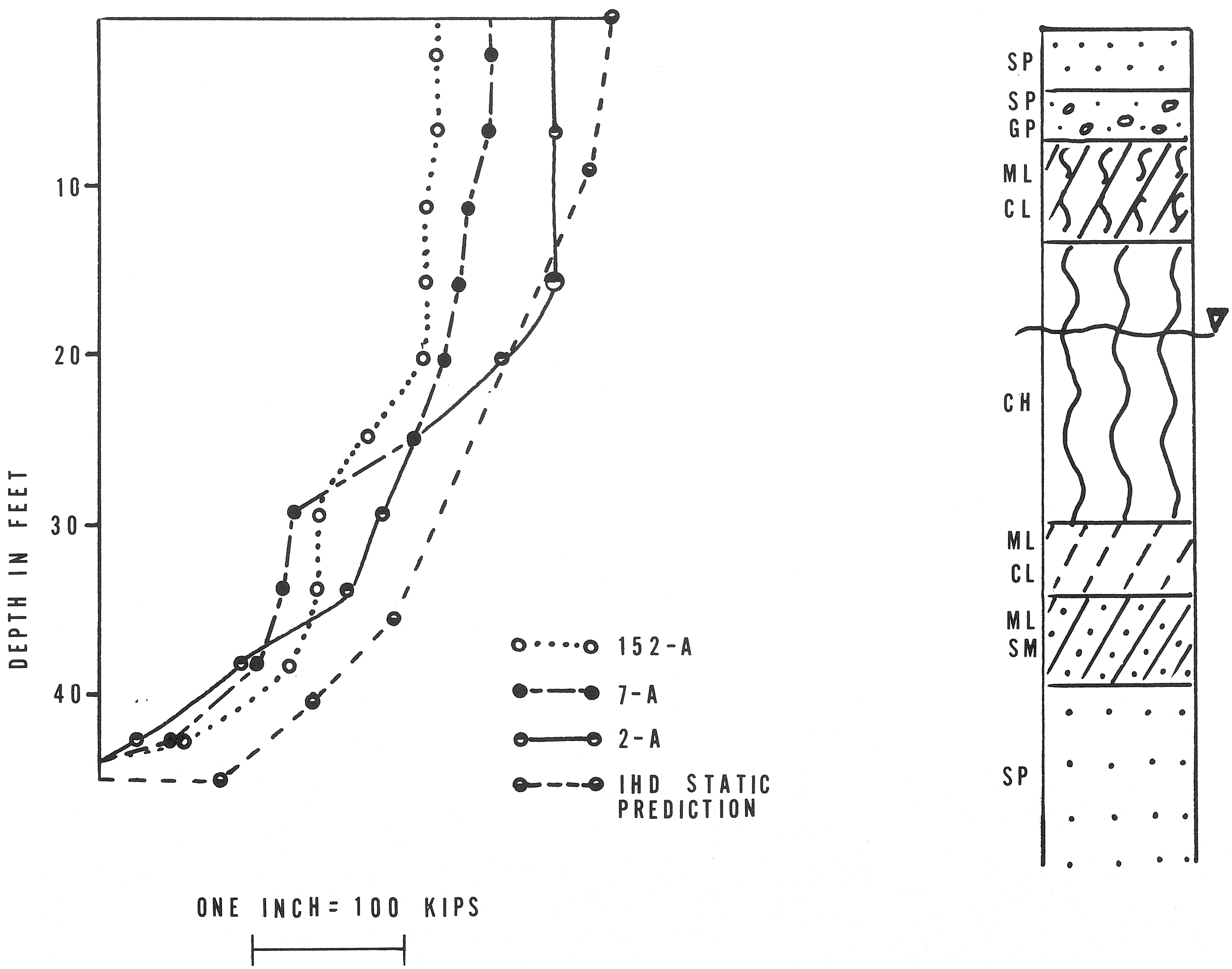

FIGURE 9: FORCES IN PILE AT PREDICTED ULTIMATE CAPACITY FOR VARIOUS DYNAMIC AND IHD STATIC PREDICTIONS

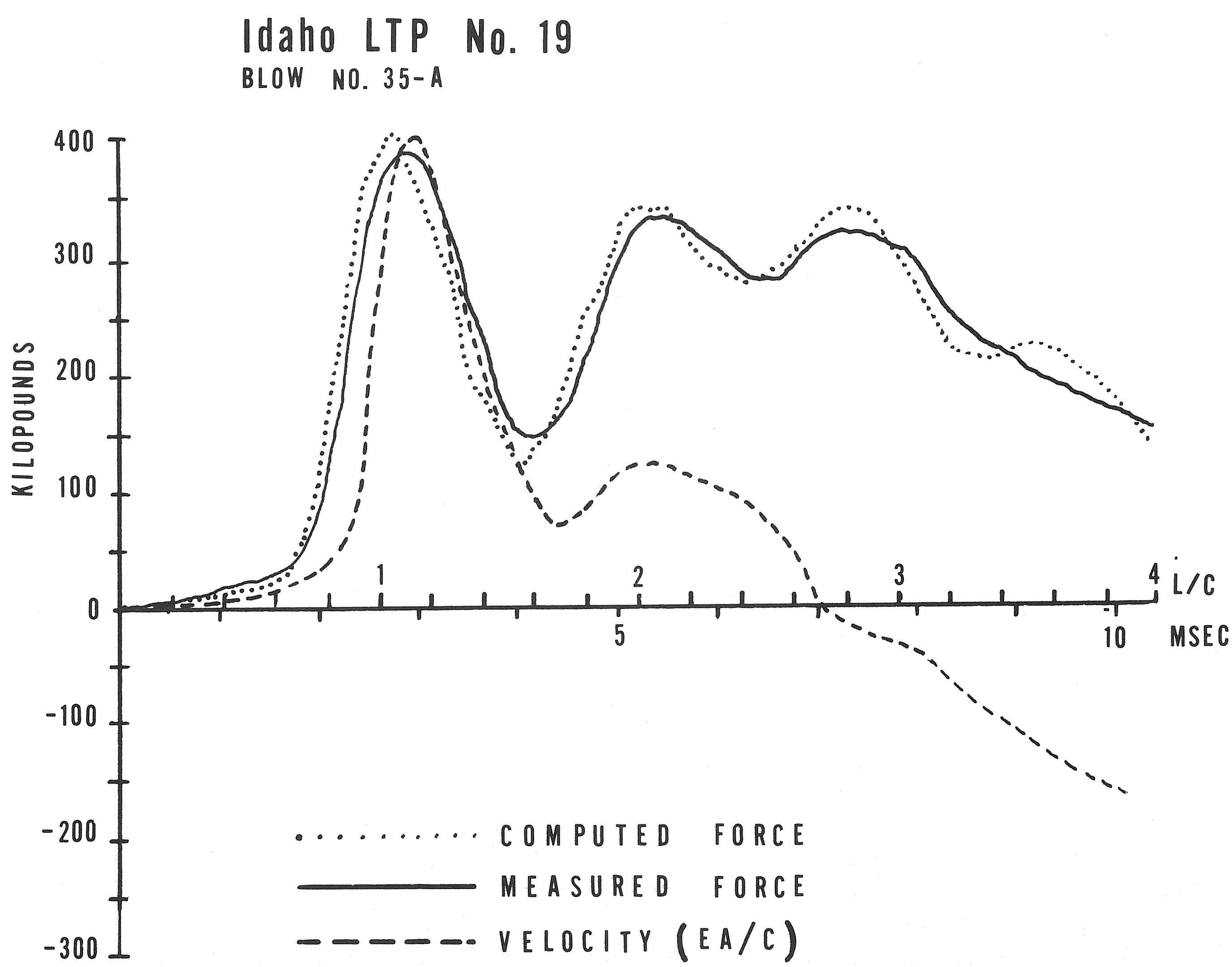

FIGURE 10: COMPUTED FORCE AND MEASURED FORCE AND VELOCITY FOR PILE NO. 19, BLOW NO. 35-A.

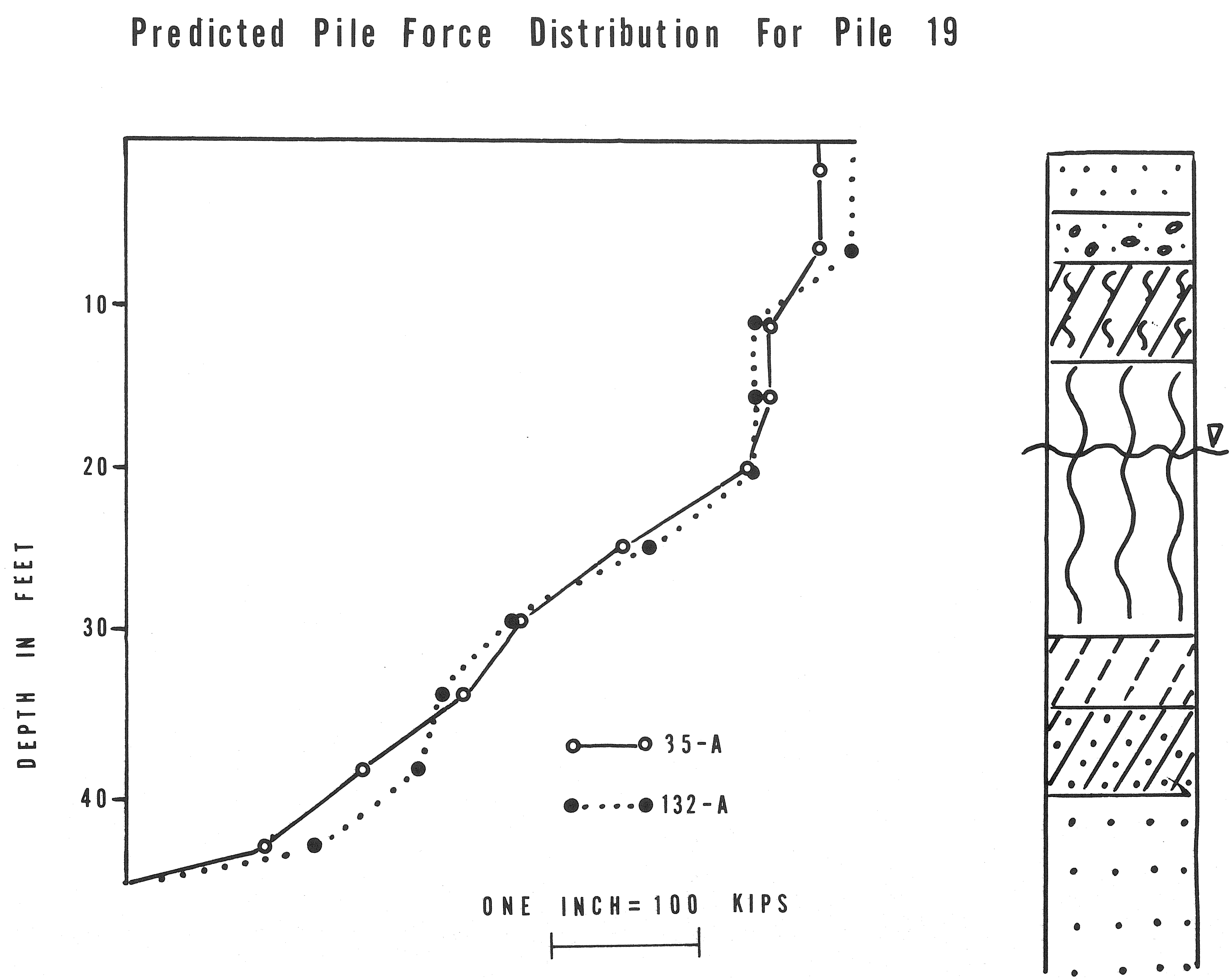

FIGURE 11: RESISTANCE DISTRIBUTION AS PREDICTED FROM TWO BLOWS FOR PILE NO. 19.

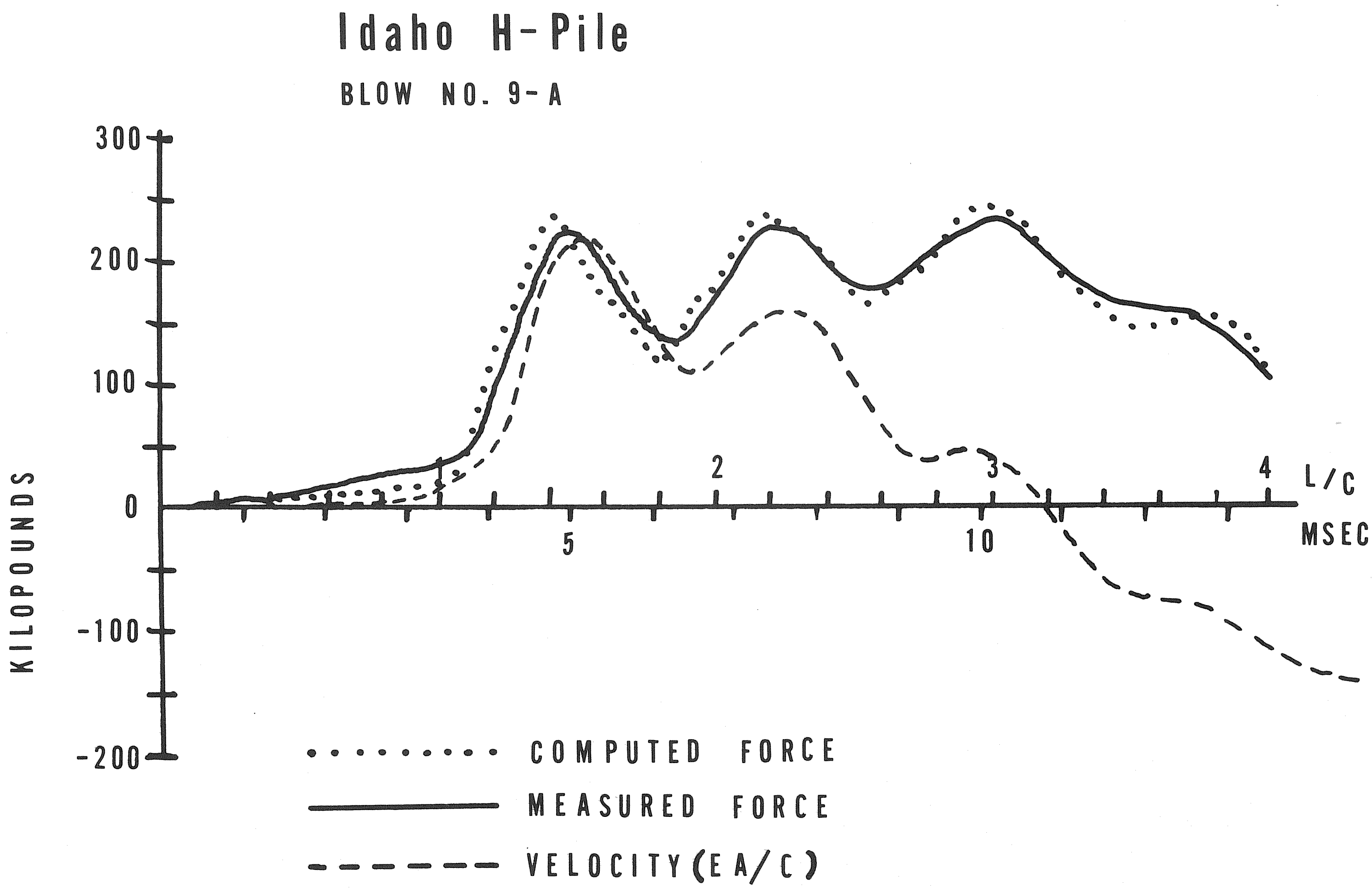

FIGURE 12: COMPUTED FORCE AND MEASURED FORCE AND VELOCITY FOR THE H-PILE (NO. 4), BLOW NO. 9-A

Predicted Pile Force Distribution For H-Pile

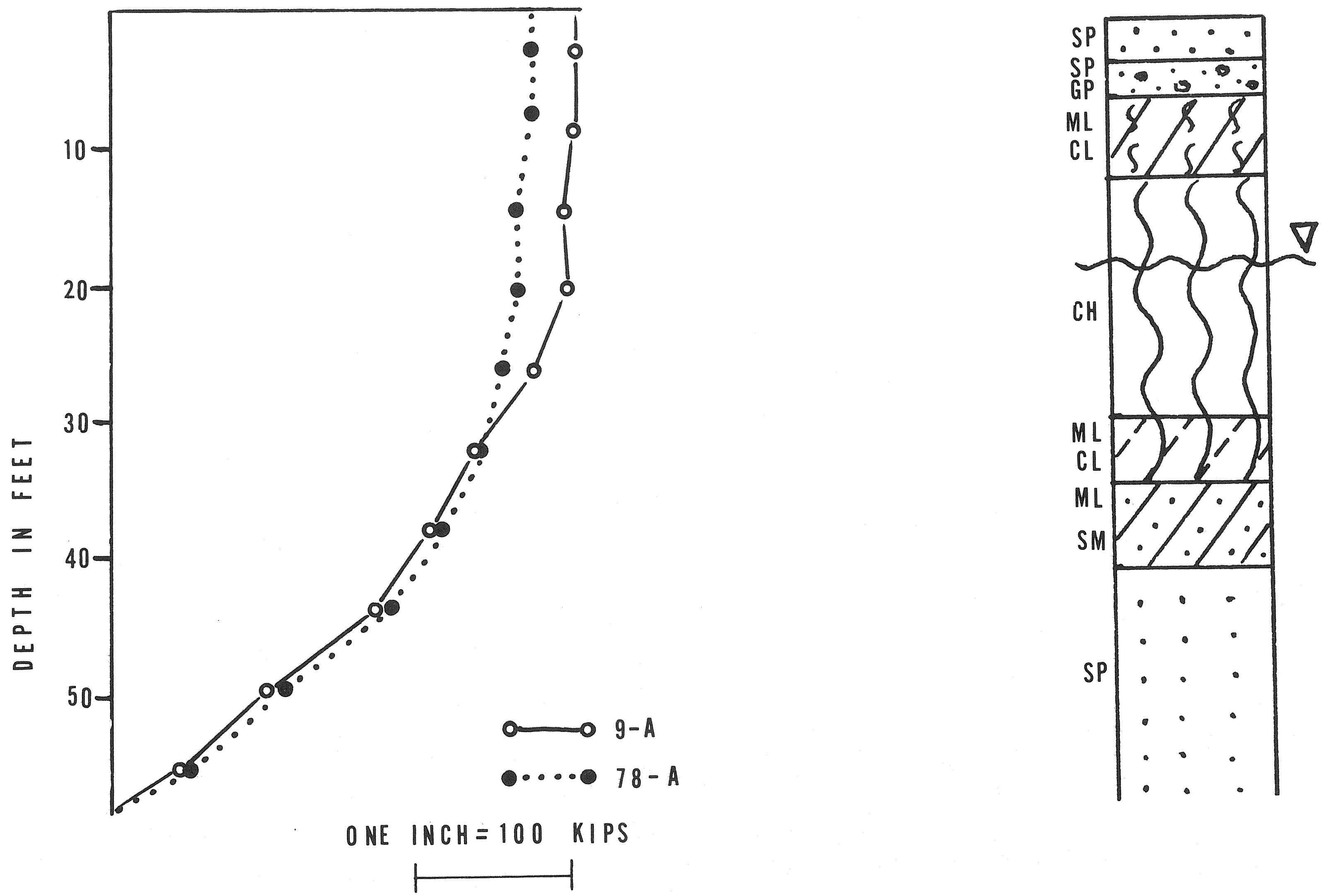

FIGURE 13: RESISTANCE DISTRIBUTION AS PREDICTED FROM TWO BLOWS FOR THE H-PILE (NO. 4).

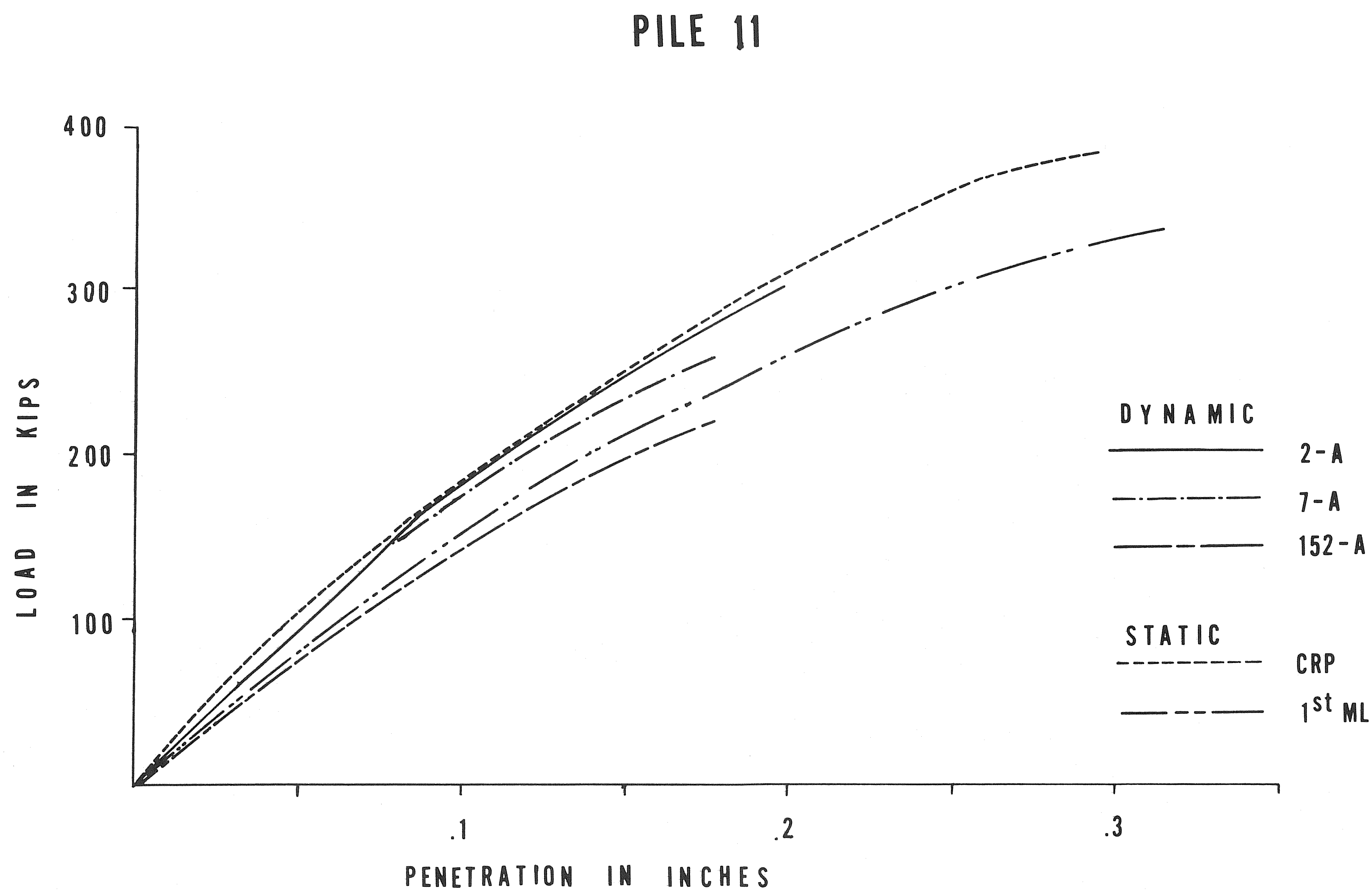

FIGURE 14: PREDICTED AND ACTUALLY MEASURED STATIC LOAD VS. DEFLECTION CURVES FOR PILE NO. 11

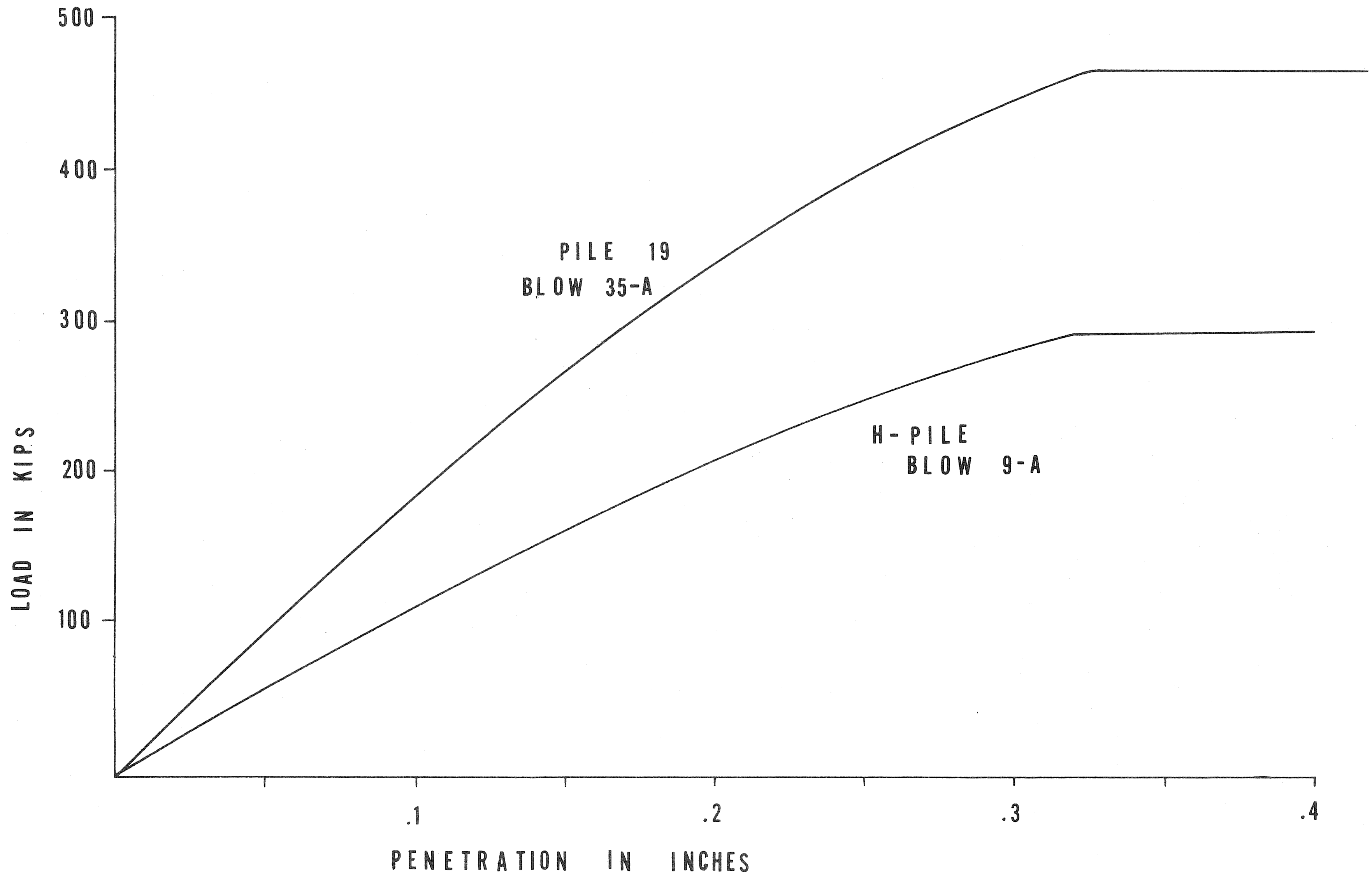

FIGURE 15: LOAD VS. DEFLECTION CURVES PREDICTED FOR PILE NO. 19 AND FOR THE H-PILE.